C5.5 Steel Girders and Beams

C5.5.2 CWPG LRFD

C5.5.2.1 General

C5.5.2.1.1 Policy overview

Methods Memo No. 184: Policy for LRFD Design
1 October 2007

See C00.

Comment: End Span Policy
14 December 2005

The office generally has followed an unwritten policy that the end span should not exceed 54% of the adjacent interior span. Origin of the policy is uncertain but apparently is the result of some study during the design of the first continuous welded plate girder bridge many years ago.

C5.5.2.1.2 Design information

C5.5.2.1.3 Definitions

C5.5.2.1.4 Abbreviations and notation

C5.5.2.1.5 References

C5.5.2.2 Loads

C5.5.2.2.1 Dead

Methods Memo No. 24: Beam Design and Bearing Design, Distribution of Dead Load 2
4 September 2001

See C5.4.2.2.1.

C5.5.2.2.2 Live

Methods Memo No. 182: LRFD Live Load Distribution for Skewed Bridges with Non-standard Rolled Steel Beams, Non-standard Prestressed Beams or Welded Plate Girders
1 July 2008

Bridge Design Manual Article 5.5.2.2.2

When beam and girder bridges meet the requirements of Article 4.6.2.2.1 of the current 2007 AASHTO LRFD Bridge Design Specifications, 4th Edition, the distribution of live load specified in Articles 4.6.2.2.2 for moment and 4.6.2.2.3 for shear should be used for design. The optional moment reduction in Article 4.6.2.2.2e shall not be used for skewed bridges.

The skew correction factor in Article 4.6.2.2.3c for shear at an end reaction only (e.g. – abutment or support under slab expansion joint), shall be used, which increases the shear live load distribution factor.

The policies above will apply to all non-standard prestressed beam, nonstandard rolled steel beam or welded girder bridges.

If you have any questions, please check with Dean Bierwagen or me. This policy should be used on any new projects that have not been completed.

C5.5.2.2.3 Fatigue

C5.5.2.2.4 Dynamic load allowance

C5.5.2.2.5 Earthquake

C5.5.2.2.6 Construction

Methods Memo No. 183: Policy Regarding Construction Loading
1 January 2008

This memo is to inform designers of information that has been added to Chapter 11.02 of the Office of Construction, Construction Manual. Because of these additions, designers may be required to do additional reviews of construction loadings on bridge projects. The following is a summary of the information that was added.

"For most bridge projects, it is assumed that construction can take place without cranes and construction equipment on the bridge. However, the Contractor will be required to submit for review and approval structural analysis by a licensed engineer when one of the following loading conditions occurs during bridge construction:

1. For bridges with weight restrictions: all vehicles and equipments exceeding the posted limit.
2. Cranes or other construction equipments that are self propelled or transported to the project site by other means and considered legal or permitted during transport if:
    a. Other components are added resulting in overall weight greater than legally allowed or granted by special permit.
    b. The operational weight including construction loads is greater than legally allowed or granted by special permit.
    c. Load distribution is altered during operation due to the use of outriggers or other devices that are not positioned over beam lines. Such use may result in localized deck overstress.
3. The use of heavy construction equipments on bridge decks with:
    a. Damaged members
    b. Critical load carrying members being replaced or repaired
    c. The presence of other construction loads including equipments and construction material in conjunction with the specified load limits in Items 4 & 5.
4. Storage greater than 300 lbs/ft^2 of construction material on the bridge over a 4 ft x 8 ft area closer than 15 ft between loads.
5. Storage greater than 60.0 lb/ft^2 of construction material on the bridge over area greater than 4 ft x 8 ft."

Until this is included in the specifications, cost of engineering analysis will be as directed by the Office of Construction. If you have any questions please check with me.

C5.5.2.3 Load application to superstructure

C5.5.2.3.1 Load modifier

C5.5.2.3.2 Limit states

C5.5.2.4 Plate girders

C5.5.2.4.1 Analysis and design

C5.5.2.4.1.1 Analysis assumptions

C5.5.2.4.1.2 Materials

Methods Memo No. 78: Charpy Requirements for Steel End Diaphragms
24 July 2003

See C5.5.2.4.2.

C5.5.2.4.1.3 Design resistance

C5.5.2.4.1.4 Section properties

C5.5.2.4.1.5 Moment

C5.5.2.4.1.6 Flanges

From 1977 until 2002 the office followed the policy of recommending a flange thickness limit of 2 inches (50 mm). The limit was stated in FHWA Notices N 5040.23 dated 16 February 1977 and N 5040.27 dated 17 August 1977. The office now uses a larger recommended flange thickness limit of 2.5 inches (63 mm) on the basis of Table 4.4 in *Bridge Welding Code, AASHTO/AWS D 1.5M/D1.5: 2002.* The minimum preheat and interpass temperature generally is the same for plates 1 ½ to 2 ½ inches (38 to 63 mm) thick.

In the 1970s the office followed a rule that the top flange area should be at least 45% of the bottom flange area. Although no explanation for the rule is available, the rule probably promoted constructibility by ensuring a certain amount of lateral stiffness for a welded plate girder. Because of the constructibility article in the LRFD specifications [AASHTO-LRFD 6.10.3], the office has rescinded the 45% rule and requires that the designer meet the constructibility provisions in the AASHTO LRFD specifications.

Methods Memo No. 103: Plate Thicknesses for Steel Bridges
16 September 2004

Recently it has been brought to our attention that steel fabricators are having difficulty obtaining steel plates with thicknesses greater then 2 inches. Currently only one mill in the nation supplies these larger plate sizes and lead time required for shipments have been unpredictable. Therefore, until the situation improves, we would like designers to limit plate thicknesses to 2 inches or less if possible. We realize that for larger span steel bridges, this requirement may not be realistic because of practical flange widths and for those projects alternatives will have to be discussed.

Methods Memo No. 131: Continuous Welded Plate Girder Butt-Welded Flange Splice Substitutions
17 August 2006

See C11.9.2.

Methods Memo No. 100: Flange Transitions in Welded Girder Bridges
30 December 2004 (Weight savings was revised on 7 July 2006.)

Current office practice is to consider a flange transition in the negative regions of welded plated girders to save material. However, a weight savings of approximately 1000 pounds (453 kg) **(Based on a May 2006 meeting with steel fabricators the weight savings was revised downward in the manual to 800 pounds (362 kg).)** per flange splice should be realized in order to justify the shop welded butt splice. See Bridge Design Manual article 5.5.1.4.1.6. Current design practice is to reduce the flange plate thickness, and maintain the same flange plate width as the thicker plate.

At welded flange splices, it is also good design practice to limit the flange cross sectional area of the thinner plate to not less than 50% of the cross sectional area of the thicker plate. This practice reduces the stress concentration at the transition area. Therefore, for all future welded plate girder bridges when welded flange butt splice transitions are used, the thinner plate shall not be less than 50% of the thicker plate.

In addition, when designing the bottom negative flange next to the adjacent positive region (field splice location), the designer should try and maintain the same approximate width compared to the positive flange plate or larger for aesthetic reasons.

References:

Myths and Realities of Steel Bridges, 1994 by AISC

Example 1
Three-Span Continuous Composite I Girder, LRFD, Third Edition
By Grubb & Schmidt

C5.5.2.4.1.7 Lateral bracing

C5.5.2.4.1.8 Shear connectors

Methods Memo No. 89: Shear Stud Lengths and Haunch Requirements for Steel Girders
26 January 2004(Manual text changed provisions of this memo in 2005 as noted in bold type.)

Because of construction problems with shear studs projecting above the top mat of steel, the following office policies have changed:

1. A minimum length shear stud of 3 ½ -inches (90 mm) may be used in the negative moment regions over the piers
2. Unless special situations warrant a maximum shear stud length of 5-inches (130 mm) should be used in the positive moment regions.

For haunch construction (allowable field haunch), the haunch thickening **(The term “thickening” no longer is used.)** will be limited by:

1. Up to ½ inch top flange embedment into the slab.
2. 3 ½ inch **(2.5 inch)** clearance between the top of slab and shear stud.
3. 2-inch minimum penetration of shear stud into the bridge deck
4. 3-inch **(4.0-inch)** maximum haunch.

For haunch design, the haunch thickness shall remain 0-inch minimum and 2-inch maximum.

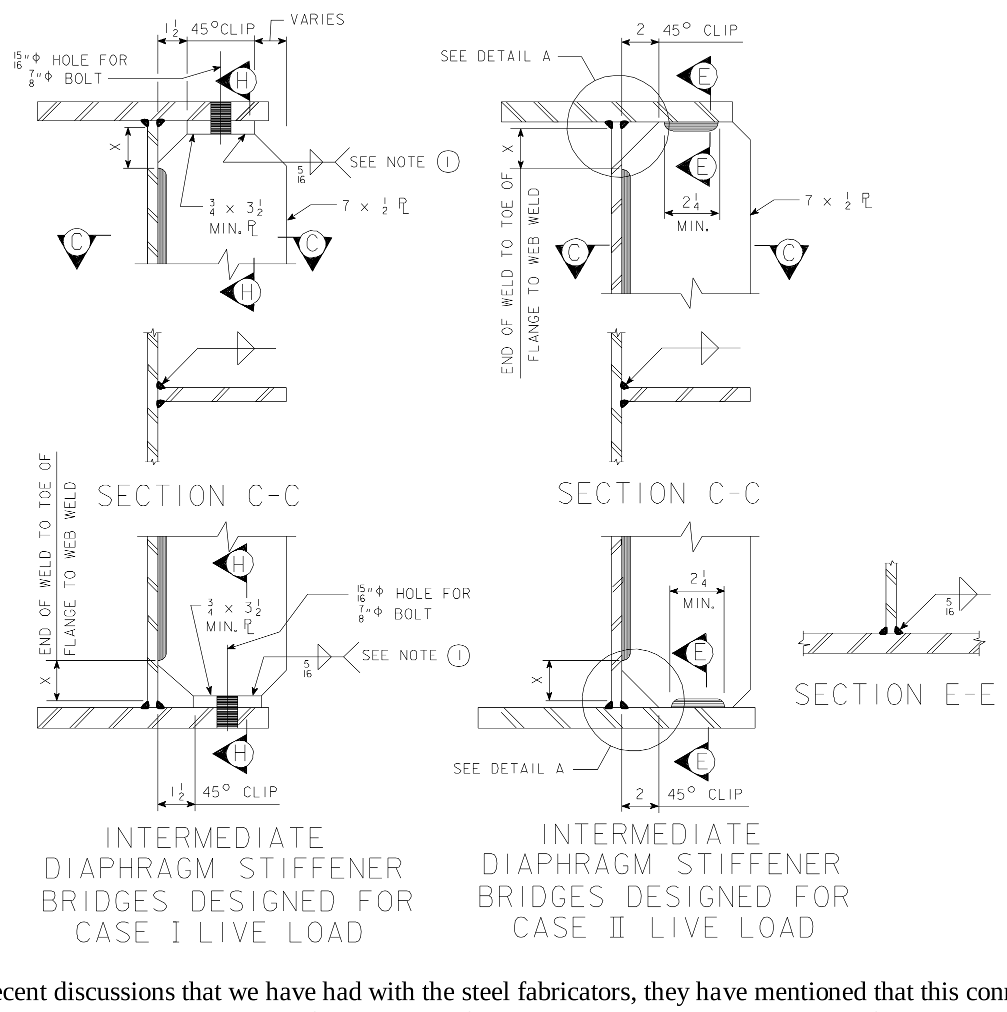
C5.5.2.4.1.9 Shear

C5.5.2.4.1.10 Web

C5.5.2.4.1.11 Stiffeners

Methods Memo No. 37: Diaphragm Stiffener Connections for Case I
7 January 2002 (The LRFD specifications do not make the Case I and Case II distinction in the standard specifications. As of July 2005 the office still prefers the welded rather than bolted stiffener, however.)

The Office of Bridges and Structures CADD standard 1021 shows the use of bolted tabs for the connection of the intermediate diaphragm stiffener to the flanges (See details below). This detail is used for the Case I live load because of the lower allowable fatigue stress (13 ksi compared to 21 ksi for the Category C weld). The current office policy is to use the bolted tab detail for all intermediate diaphragm stiffener connections when Case I stress cycles are used for fatigue (AASHTO 10.3.2A).



In recent discussions that we have had with the steel fabricators, they have mentioned that this connection is extremely expensive because of the amount of labor it takes to complete. Because of the high cost, we would like to minimize the use of the bolted tab connection by revising our current office policy as follows:

1. During design, check the fatigue stresses at the diaphragm connections to see if the stress range would allow the use of the welded connection (Case II details).

2. If the stress range exceeds the allowable for the Category C welded connection, then consider increasing the flange thickness to decrease the stress range. By designing the flange thickness for the lower fatigue stresses, the standard welded connection that is shown in the Case II live load can then be used.

   Note: This will generally be in the positive bending regions where the diaphragm welded stiffener connection controls the fatigue stress for the bottom tension flange. In the negative regions where tension stresses are on the top flange, the shear connectors along with the welded diaphragm connections may control the fatigue stresses.

3. A cost comparison should be made between the cost of bolted tabs and the additional cost of steel material. If the additional cost of steel for the larger flange thickness become excessive, then the Case I detail shall be used. The locations for the Case I details will need to be shown in the framing diagram. For estimating costs, assume $350.00 per stiffener to provide the bolted tab detail and $.30 per pound of steel.

Using a thicker flange plate rather than the connection should reduce the overall cost of the bridge because of the savings in labor. This policy change is for straight girders only. If you have any questions check with your section leader.

C5.5.2.4.1.12 Deflection and camber

C5.5.2.4.1.13 Welded connections

C5.5.2.4.1.14 Bolted connections

C5.5.2.4.1.15 Fatigue

C5.5.2.4.1.16 Diaphragms and cross frames

At the time of the January 2006 Bridge Design Manual update the standard cross frames used by the office were redesigned to meet AASHTO LRFD specifications (even though the manual still was based on the AASHTO standard specifications). The following is a summary of the AASHTO and AISC changes that affected the redesign.

- Single angles connected with bolts and welds no longer are permitted to use K = 1.0; K must be 0.75 [AASHTO-LRFD 4.6.2.5]. This 2005 AASHTO LRFD change is in the direction of conservatism and makes published cross frame examples obsolete. Because many cross frame members are at the KL/r<=140 limit for compression members, the change has a significant effect on member size.
- Webs of rolled shapes (and presumably stems of tees) no longer are permitted to be 0.23 inches thick; they now must be 0.25 inches thick [AASHTO-LRFD 6.7.3]. This 1998 or earlier AASHTO LRFD change from the standard specification is in the direction of conservatism and makes our use of WT 4x9 (with a stem thickness of 0.230 inches) obsolete. We need to use at least a WT 4x10.5.
- Outstanding legs of angles no longer are permitted to have a maximum b/t ratio of 16; they now must have a ratio of 15.89 for A36 steel or less for higher grades of steel [AASHTO-LRFD 6.9.4.2]. This 1998 or earlier AASHTO LRFD change from the standard specification is in the direction of conservatism and requires thicker angle legs in some cases.

- For relatively thick angle legs, AISC permits an increase in flexural capacity from 1.25M_y to 1.50M_y. This change in the 2000 AISC single angle specification reduces conservatism in angle capacity for angles with relatively thick legs.

Where diagonals cross, there is the question of whether the crossing connection can be considered a brace point for out-of-plane buckling. Two papers in AISC's *Engineering Journal* give justification for a brace point if one of the two diagonals is in tension. If both diagonals are in compression, however, the crossing connection is not a brace point for out-of-plane buckling.

Critical cases for design of the cross frames were the following:

- Pier frame: diagonal in completed structure, Strength III for wind, with center brace point
- Intermediate frame: diagonal during construction, Strength III for wind, without center brace point; diagonal during construction, Strength I for deck pour, without center brace point
- Intermediate frame: strut during construction, Strength I for deck pour

C5.5.2.4.1.17 Horizontally curved superstructures

C5.5.2.4.1.18 Additional considerations

Methods Memo No. 65: Limit Steel Girder Lengths Between Field Splices 120 ft
13 May 2002

When designing steel girder bridges, try to limit the shipping lengths to a maximum of 120 feet (36.5 m). With segments less than 80 feet, the designer should consider combinations with other section to reduce the number of field splices keeping in mind the maximum shipping length and dead load inflection locations. This policy should allow more local fabricators the option of bidding on the projects.

C5.5.2.4.2 Detailing

Methods Memo No. 151: Steel Bridges Providing Tension and Compression Flange Designation 22 March 2006 (The reference to Article 2408.02, J, 1, is for the 2009 Standard Specifications, revised from 2408.15, A, 2, (c). ~ 16 June 2009)

Proper welding and weld inspection of plate girders and rolled steel beams require identification of the tension and compression flanges. The standard flange butt-welded splice detail and intermediate stiffener detail shown on OBS SS 1021 (M1021) included with plate girder bridge plans refers to tension and compression flanges. Stiffener-to-web welds discussed in the Iowa DOT standard specifications [IDOT SS **2408.02, J, 1**] also refers to tension and compression flanges.

For plate girder bridges, the designer shall provide on the "Girder Elevation" detail the location of tension and compression areas for both the top and bottom flange of the girder. The location of tension and compression flanges along the length of the span is to be based on the dead load inflection point of the member. These locations will normally correspond with the locations of the bolted field splices. The rolled beam standards (OBS SS RS40-BM1-04 to RS40-BM8-04) currently indicate this information on the beam elevation view.

This policy shall be followed for all plate girder (or non-standard rolled steel beam) bridges not yet turned in.

Methods Memo No. 78: Charpy Requirements for Steel End Diaphragms
24 July 2003

After discussions with the Materials Office, it was found that the standard specifications are not clear on when Charpy toughness is required for girder stiffeners. The question was raised whether Charpy toughness testing is required for stiffeners that connect end diaphragms under expansion joints. Because the stiffeners are part of the system that transfers load from the floor beam to the longitudinal girder, Charpy toughness testing is required. To help clarify the issue, the following note should be added to steel bridge plans that have end diaphragms under expansion joints.

> In addition to the requirements of 4152 of the Standard Specifications, Charpy V-notch requirements shall also apply to the stiffeners connecting the floor beam diaphragms to the girders at all expansion joint locations.

Methods Memo No. 71: Note on Option of Welding Studs in the Field
18 June 2002

The following plan note is no longer valid: "Stud shear connectors are to be welded in the shop or in the field at the locations shown on the design plans or the approved drawings." Due to a ruling from OSHA, the contractors are not allowed to install shear studs in the shop unless specific safety procedures are followed. Since we are not in charge of enforcing OSHA regulations, it is best not to tell the contractor how to meet these regulations thus reducing our exposure to liability. We will no longer include this note on our plans.

Methods Memo No. 164: Stiffener Clearances
4 September 2007

The following policy change has been made for minimum clearances for placing transverse stiffeners next to shop-welded splices for welded plate girder bridges. The minimum spacing in the Bridge Design Manual, “5.5 Steel Girder” article “5.5.1.4.2 Detailing” has been revised as shown.

> “Shop welded flange splices shall be at least 6 inches (150 mm) from a stiffener, 6 inches (150 mm) from a web splice, and 4 inches (100 mm) from a shear connector. Web splices shall be at least 6 inches (150 mm) from a stiffener. Splices shall not interfere with other bridge components.”

In addition, the standard CADD sheets 4305, 4308, 4309, and 4310 have been revised by adding the following note:

SHOP WELDED FLANGE SPLICES SHALL BE A MINIMUM OF 6 INCHES FROM A STIFFENER, 6 INCHES FROM A WEB SPLICE, AND 4 INCHES FROM A SHEAR CONNECTOR. WEB SPLICES SHALL BE A MINIMUM OF 6 INCHES FROM A STIFFENER. SPLICES SHALL NOT INTERFERE WITH ANY OTHER BRIDGE COMPONENTS. ALL SHOP WELDED BUTT SPLICES SHALL BE SHOWN ON THE SHOP DRAWINGS AND SUBJECT TO APPROVAL BY THE ENGINEER.

This change was requested by the fabricator and is based on what is recommended nationally by AASHTO/NSBA Steel Bridge Collaboration at the web site noted below:

http://www.steelbridges.org/collaboration_pages/AASHTO%20Docs/DDPG-1%20AASHTO.pdf
If you have any questions on the updated sheets, please check with me or Dean Bierwagen.

C5.5.2.4.3 Shop drawings

Methods Memo No. 38: Review of Shop Drawings—Steel Structures
24 January 2002

As stated in the Iowa Department of Transportation, Standard Specifications for Highway and Bridge Construction, Series 2001, “…The Contractor shall understand that the Contracting Authority’s review of working drawings submitted by the Contractor covers only requirements for strength and arrangement of component parts…” Furthermore, the Iowa Department of Transportation, Office of Bridges and Structures uses the following stamp on all Shop Drawings:

REVIEWED BY OFFICE OF BRIDGES AND STRUCTURES IOWA DEPARTMENT OF TRANSPORTATION
Reviewed in Accordance With Current Policies JUL 25 2001 ☐ NO EXCEPTIONS TAKEN ☐ MAKE CORRECTION NOTED (NO RESUBMITTAL NEC.) ☐ AMEND AND RESUBMIT ☐ FOR DISTRUBUTION BY_________________

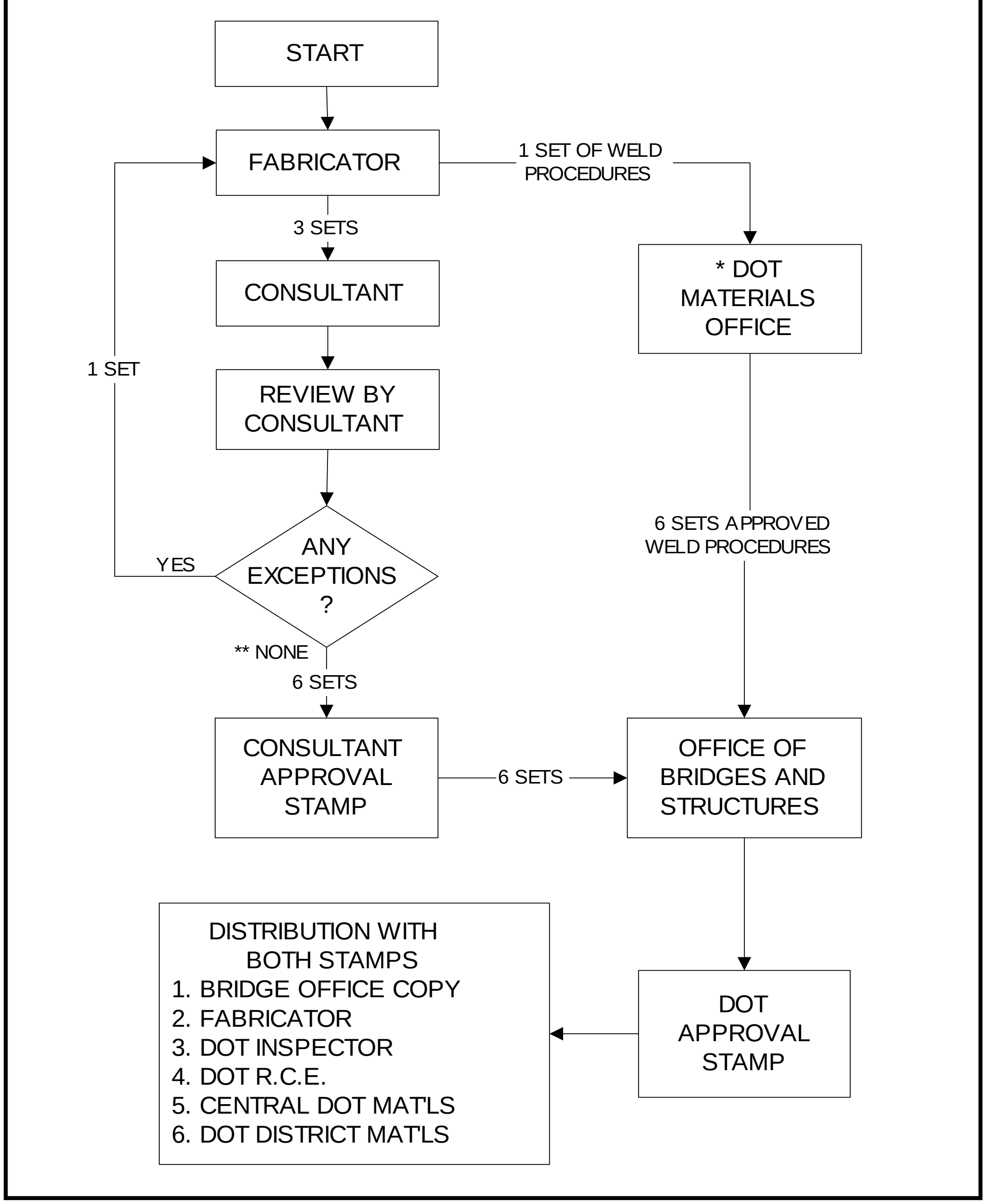
Shop drawings submitted by fabricators and suppliers should be checked to ensure that the structural adequacy of the design is maintained as detailed on the original design drawings. The extent of the Shop Drawing check will vary with the individual design. On curved and complex designs, the extent of the Shop Drawing check shall be discussed with the Section Leader or the Consultant Coordination Section prior to review.

A detailed check of the dimensions is not required. However, depending upon the level of difficulty or complexity of the structure, a “spot check” of critical locations may be performed. The intent of these guidelines does not preclude sound engineering judgment?

Changes from the contract plans or specifications, regardless of magnitude, should not be allowed unless they have been documented previously as acceptable or have been approved by the Section Leader or the Consultant Coordination Section.

Review and oversight of projects involving structures designed and developed by consultants is the responsibility of the Consultant Coordination Section. The following procedure shall be followed for all Shop Drawings reviewed by consultants:

Flow Chart of Structure Fabrication Drawings Reviewed by Consultant



* Communications between the fabricator and the DOT Materials Office is a separate approval process for welding procedures, QC, QA, etc.
** Including "APPROVED AS NOTED".

Note: Consultant may require additional sets of drawings.
Six sets of drawings may be submitted initially if few changes are anticipated.

As a means of establishing uniform practice and avoiding omissions, the following guidelines are provided for the “standard-type” wide flange and welded plate girder bridge.

The following items shall be checked:

1. Verify that all material shown on the Shop Drawings conforms to the size, thickness and material type shown on the contract plans.
2. Check the flange and web plate cutoffs
3. Check the location of main beam and girder splice locations, and details of connections not dimensioned on the contract plans
4. Check number and approximate location of diaphragms.
5. Check the number and size of bolts (diameter only – not length) in all connections
6. The number of shear studs
7. Check the size of all welds and welding details
8. The finish on bearing assemblies
9. The amount of camber and the camber diagram configuration should conform to the contract plans

The following items do not need to be checked:

1. The length of members or components
2. The location and gauge of holes
3. The camber diagram ordinates (girder lay down dimensions)
4. Web cutting diagrams
5. Attachments to expansion plates
6. Bill of Materials

Appendix for obsolete and superseded memos

Methods Memo No. 104: LRFD Implementation for Steel Bridge Design
3 June 2005

As part of the Office of Bridges and Structures implementation plan for the AASHTO LRFD specification, the following design policy has been adopted. All new steel girder bridge projects (excluding curved steel bridges) designed in house shall use the 2004 AASHTO LRFD 3rd Ed. specification. LRFD design of steel bridges by consultants will be determined on a case-by-case basis.

The new policies include the superstructure for welded plate girders (barrier, deck, and girder). Updated standard slab cross sections for steel bridges will be available in the future for the English and metric standard roadway cross-sections. The 3 span rolled steel standards will continue to use the AASHTO Standard Specifications and will be updated in the future for LRFD. The substructure designs will continue to follow the 2002, AASHTO Standard Specifications, 17th Ed.

Examples and MathCAD files are available for use by office personnel and consultants. Questions regarding the design methods or specifications shall be directed to John Neiderhiser or Ahmad Abu-Hawash.